DESIGN AND CONSTRUCTION OF THE

BURLINGTON-BRISTOL BRIDGE

ACROSS

THE DELAWARE RIVER

by

ELLIS E. PAUL

B.S. in C. E. University of Kansas 1922

Submitted to the Department of Civil Engineering and the Faculty of the Graduate School of the University of Kansas in partial fulfillment of the requirements for the degree of Civil Engineer.

Approved by:

H. A. Rice
Instructor in Charge

H. A. Rice
Head of Department

April 24, 1931

PREFACE.

The demands of civilization, with its constant desire for progress, betterment and speed, has always been directly responsible for new inventions, new methods and new construction. The improvements of this country's highways has been responsible for the progress of the motor car industry, and this progress along with the public's desire for speed and comfort has demanded more roads and more arteries leading to all parts of the nation. It has opened new sections of land to the traveling public, sections never before visited except by local residents and occasional business men. This has naturally resulted in increased business, thereby developing a large cross state and interstate bus and truck traffic.

Mother Nature, in the form of streams and rivers has placed a large number of natural barriers in the path of this progress. To overcome these barriers, it has been necessary to construct bridges of various types and magnitude and it is the purpose of this paper to discuss one of these bridges, unique in size, in design and in construction.

In the preparation of this paper, it has been intended that only the more important features governing the design and construction of the bridge, be given. These subjects are covered briefly yet fully, and in such a manner as to be understandable and interesting. No attempt has been made to discuss or use any of the technical parts of the design. It has been prepared with the idea that the readers will be mainly undergraduates whose interest will lie wholly in reviewing the subject as a news item and not as a text book. It is hoped that the reader will obtain a general idea as to the advantages of the vertical lift bridge, the features governing its design and construction, its method of operation and the possibilities of its use in meeting extreme conditions.

Burlington, N.J. E.E.P

March, 1931.

TABLE OF CONTENTS.

LIST OF ILLUSTRATIONS

DESIGN AND CONSTRUCTION OF THE BURLINGTON-BRISTOL BRIDGE ACROSS THE DELAWARE RIVER.

For many years the advantages of the vertical lift bridge have been apparent over other types of movable bridges. This has been demonstrated by the increasing favor this type of bridge has found with various engineers. It has almost been universally adopted by the railroads in this country. In the past fifteen years certain cities embodying the most congested highway and river traffic have adopted the vertical lift bridge as being the most economical and advantageous in overcoming their traffic conditions. A few of these cities include Portland, Oregon; Jacksonville, Florida; Mobile, Alabama; Cleveland, Ohio; Norfolk, Virginia; Tacoma, Washington; Louisville, Kentucky; Jersey City, New Jersey; and New York City. In addition to these the United States Government, after competitive designs of all types of movable bridges, chose to erect five vertical lift bridges over the Chesapeake and Delaware Canal. In the past three years the Canadian Government, also after competitive designs, has completed seventeen vertical lift bridges over the Welland Ship Canal. Certainly such a preference must be justifiable, and a few of the advantages of this type of bridge will be listed. They are:

1. Economy in construction, both in first cost and in operation.

2. Rapid operation at low cost.

3. Determinate stresses.

4. Need not be opened full height for small vessels, thereby saving time and insuring low operating cost.

5. Adaptable to skewed channels.

6. Adaptable to a future change of elevation on grade.

7. Rigid under traffic.

8. Economically adapted to construction on grades.

9. Paved floor. Track construction and maintenance are very simple and economical.

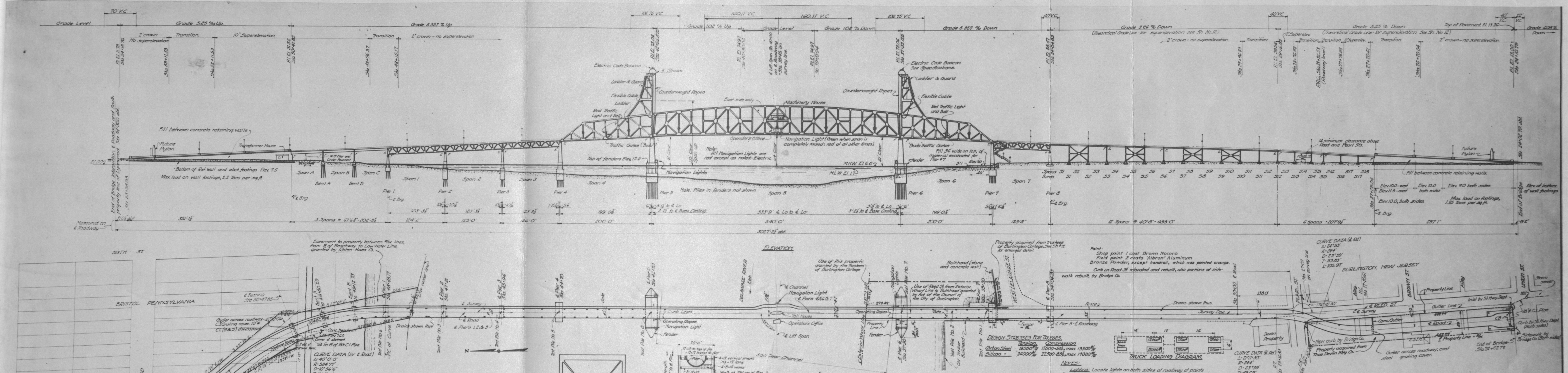
10. Economy in length.

11. Small exposed area to wind, thereby eliminating serious delay or interruption in operation during storms.

12. Mechanically and structurally advantageous.

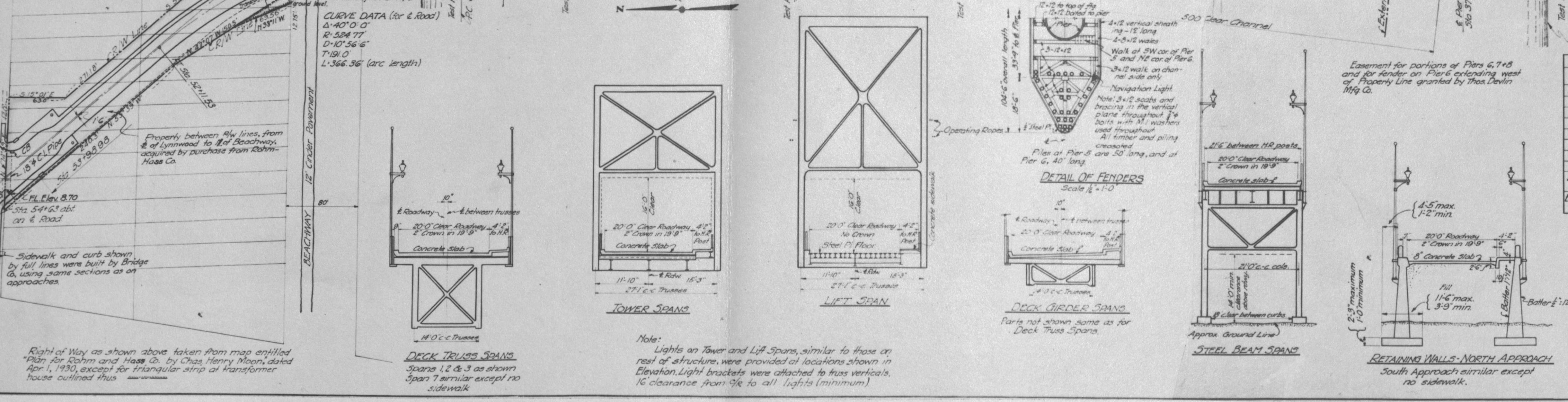
13. Saving in time of operation due to elimination of locks and end wedges.

A number of novel and original features, both in design and construction, tend to make the new vertical lift bridge between Burlington, New Jersey, and Bristol, Pennsylvania, one of the most interesting structures to be constructed in the year 1930. Chief among these are the long vertical lift span, its light weight floor system and its method of erection. This lift span, 533'-9" center to center of end bearings, is the longest of its type ever to be built; being about 150 feet longer than the next longest lift span; and as far as records show, is the longest movable span in existence.



Paint:
Shop paint 1 coat Brown Nocoro
Field paint 2 coats "Albron" Aluminum Bronze Powder, except handrail, which was painted orange.

Curb on Reed St relocated and rebuilt, also portions of side-walk rebuilt, by Bridge Co.



Right of Way as shown above taken from map entitled "Plan for Rohm and Haas Co. by Chas. Henry Moon, dated Apr. 1, 1930, except for triangular strip at transformer house outlined thus

Note: Lights on Tower and Lift Spans, similar to those on rest of structure, were provided at locations shown in Elevation. Light brackets were attached to truss verticals.

DESIGN STRESSES FOR TRUSSES.

	Tension.	Compression
Carbon Steel	16000#/□	15000-50 l/r, max 13500#/□
Silicon "	24000#/□	22500-85 l/r, max 19000#/□

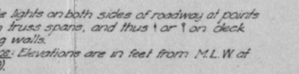
FINAL DATA ON PIERS AS BUILT.

Pier No.	Size of Base.	No. of Piles	Elev. top of Base.	Elev. bott. of Base	Av. Elev. bott. piles
1	10'×25'	24	+1.0	-8.0	-20.9
2	10'×25'	24	-2.0	-26.0	-46.0
3	10'×25'	27	-2.0	-26.0	-48.3
4	16'×40'	68	-2.0	-26.0	-46.9
5	25'×61'	163	-10.0	-30.0	-65.1
6	25'×61'	160	-10.0	-30.0	-60.9
7	16'×40'	65	-2.0	-15.0	-33.5
8	2-10×15'	none	+10.0	+7.0	—
Bent A	W 8'-7"×10'-6" E 8'-10"×10'-4"	7 5	+4.0 +4.0	+1.0 +1.0	-13.8 -17.0
Bent B	W 8'-11"×9'-1" E 9'-3"×9'-9"	5 6	+4.0 +4.0	+1.0 +1.0	-18.3 -17.2

FINAL DATA ON PEDESTALS S1 TO S15, INCLUSIVE.

Pedestal No.	Size of base	Depth of base	Elev. bott of base	Elev. bott of steel
S1-S4, inc.	6'-0"×7'-9"	2'-0"	10.3	16.8
S5	6'-0"×7'-9"	2'-0"	10.5	17.0
S6	6'-0"×7'-9"	2'-0"	10.7	17.2
S7	6'-0"×7'-9"	2'-0"	10.9	17.4
S8	6'-0"×7'-9"	2'-0"	11.1	17.6
S9	6'-0"×7'-9"	2'-0"	11.3	17.8
S10	6'-0"×7'-9"	2'-0"	11.5	18.0
S11-S12	6'-0"×7'-9"	2'-0"	11.5	18.0
S13-S14	8'-0"×8'-0"	1'-6"	11.5	18.0
S15	7'-0"×7'-0"	1'-6"	11.5	18.0

Note: No piles in Pedestals S1 to S15.



TRUCK LOADING DIAGRAM.

NOTES:

Lighting: Locate lights on both sides of roadway at points thus ✻ on through truss spans, and thus ⸸ or ⸸ on deck spans and retaining walls.

Datum Reference: Elevations are in feet from M.L.W. at Marcus Hook (1879).

Live Loads

For Floor and Beam Spans: 15 ton truck with 15 ton trailer on each 10 ft. traffic lane. See Diagram above.

Sidewalks: 100#/□'

Trusses and Girders: On roadway surface truck loading as for floor system or 2-10 ft. lanes of traffic, each lane consisting of a concentrated load of 21,000# followed or preceded or both by a uniform load of 450# per lin. ft. On sidewalk surface a uniform load in pounds per sq. ft. equal to (80-0.125L)(1-0.025W) where L = length in feet of sidewalk loaded to produce maximum stress and W = clear width of sidewalk in feet. Sidewalk load shall not be less than 20#/□'.

Impact Loads: $I = \frac{L+250}{15L+300}$ for highway loading but not to exceed 30%, except 60% for end floorbeams, floorbeam hangers, columns supporting floorbeam concentrations and all floorbeam connections. L = length in feet loaded to produce maximum stress and I = impact in % of live load. No impact for sidewalk live load.

Height Scale: On northeast and southwest columns paint a height scale with elevation 6.8 as zero. Scale to start at approximately 60 ft. Every foot shall be indicated; every fifth foot by a large mark, and a neatly made figure about 18" high painted for every 10 ft. of height.

Paint shall be of Nobrac or similar approved paint and two coats shall be applied, of colors which will contrast with the structure.

Piles in fenders were creosoted; all other piles used were untreated timber. Piles in piers and bents were driven to safely sustain a minimum load of 40 Tons per pile.

CURVE DATA (℄ Rd.)
Δ = 21°11'30"
R = 244'
D = 23°39'
T = 45.65'
L = 90.25'

LIFT BRIDGE DESIGNED BY
ASH-HOWARD-NEEDLES & TAMMEN
CONSULTING ENGINEERS
KANSAS CITY NEW YORK
PATENTED IN U.S. 5-28-'12, 1-7-'13, 7-17-'17, 4-2-'18, 11-26-'18, 2-24-'20, 2-15-'25
PATENT NO. 1671693, Others Pending
IN CANADA 2-1-'10, 3-8-'10, 9-7-'12, 9-17-'12

Patent Plate:

Provide one Bronze Patent Plate about 10"x16" with moulded border.

Letters to be Standard Tablet Letters, projecting 1/8" from background. Use lower case where indicated. Face of letters smooth. Background of Stippled effect.

A rubbing of pattern to be furnished for approval of engineers before making casting.

Bolt plate to inside face of front tower column about 5'-0" above top of sidewalk, using concealed bolts.

Revision	By	Ckd	Date
Added Clearance in Span A, North Abutment & Pier South Line of Beachway, Revised R/W Line	J.K.	J.K.	10-31-30
As built	AR		3-3-31

LOG OF PIER EXCAVATIONS.

Pier # Elev. to Elev. — with materials excavated between elevations shown

1. +4.0 M -8.0 (Mud varying in stiffness toward bottom.)
2. +4.3 M -13.3 FS+M(FG) -21.0 TYC -22.5 S+G+Cobbles (YC) -26.0
3. -8.5 FMS -23.5 CS+G (Cobbles+C) -26.0
4. -10.2 FGS+M -17.2 SM+S+D -23.2 S+G+C+B -26.2 (Clay prominent at -26)
5. -17.5 S+G -30.0
6. -8.5 S+G -18.0 C -30.0
7. +4.5 S+G+B -15.0
8. +15.0 FS+L +9.0 S+B +7.0

C = Clay. (YC = Yellow; TYC = Tough Yellow)
G = Gravel. (FG = Fine; CG = Coarse)
S = Sand. (FS = Fine; CS = Coarse; FMS = Fine, Muddy; GS = Gray)
M = Mud. (SM = Soft)
L = Loam. B = Boulders. D = Drift.

Approved: Jonathan Jones 4-25-30
Chief Engineer
Burlington-Bristol Bridge Co.

BURLINGTON - BRISTOL BRIDGE

ACROSS THE DELAWARE RIVER
BURLINGTON, NEW JERSEY - BRISTOL, PENNSYLVANIA
FOR THE
BURLINGTON-BRISTOL BRIDGE COMPANY

GENERAL PLAN AND ELEVATION

Scale 1"=30' & 1"=10'
Drawn by HJD Date 6-4-29
Traced by HJD Date 6-5-29
Checked by G.A. Date 4-23-30

Ash-Howard-Needles & Tammen
Consulting Engineers
Kansas City New York

SHEET NO. 1

The bridge structure itself runs generally north and south and is slightly more than 3000 feet long; but with approach roads connecting the two State Highway systems, the project is approximately two miles in length. The connection in New Jersey is with the State Highway route #25 which is the main route between New York, Trenton and Camden, while that in Pennsylvania is with the Bristol pike with extensions to the Lincoln Highway joining New York, Trenton, Philadelphia and points south.

Figure #1 shows the general elevation of the structure, which consists of earth fill between retaining walls, deck girder, deck truss, beam and through truss spans. The roadway is of concrete except for that part on the lift span where steel traffic plates are used for the entire covering. The structure is modern and fireproof and has a 20 foot clear roadway providing for two lanes of traffic which was thought to be sufficient, since it provides a roadway width equal to or greater than the highways with which it connects. The question of a wider roadway was fully discussed but abandoned as being uneconomical to meet the demands of traffic at this location. The roadway is adequately lighted throughout with ornamental street lights and provision is made to care for pedestrian traffic by the use of one 4 foot sidewalk along the east side and with entrance by stairs at the south river bank.

The requirements of navigation are cared for by the provision of a 500 foot clear horizontal channel with a minimum vertical clearance above mean high tide of 61 feet at the fender lines and 64 feet at the center of the span. This vertical clearance with the span closed is increased to 135 feet at the fender lines with the span fully opened. To meet the navigation requirements it was necessary to provide one horizontal opening of 500 feet with 61 feet vertical clearance and another opening of 250 feet with 135 feet vertical clearance. Due to the narrowness of the channel and its closeness to the south shore, it was necessary to combine both the above requirements into one long movable span, instead of using one long fixed span and one shorter movable span.

DESIGN LIVE LOADS.

Figure 1, showing the general elevation of the structure, also gives a table listing all design loads used on the floor system, the sidewalks, the trusses and girders, as well as the various impact loads on the above classifications.

Wind Loads. 30 pounds per square foot on exposed areas, consisting of one floor, two handrails and two trusses. This was used on the entire structure including the lift span in the closed position. In the open position, a load of 15 pounds per square foot was used on the lift span and towers. In the design of the bottom chord members of the trusses a 10 pound wind was used in conjunction with live load.

Unit Stresses.
Allowable axial compressive stress-
Carbon.15000 - 50 l/r Maximum 13500 #/sq.in.
Silicon22500 - 85 l/r Maximum 19000 #/sq.in.
Allowable axial tensil stress-
Carbon 16000 #/sq.in.
Silicon24000 #/sq.in.

The use of silicon steel was found to be economical only in the lift span where a saving in weight was necessary. The top and bottom laterals as well as all truss members, except counter web members and hangers of this span were of silicon steel. All other steel was carbon. An examination of the above unit stresses, along with a later discussion regarding the effect of weight on the entire structure, will easily disclose the cause and effect of this economy.

PIERS, FOUNDATIONS AND FENDERS.

Work on the piers and fenders was begun about the first of May and was completed the first of September, 1930, in the remarkably short period of four months. This was made possible by coordination between the sub-contractor and the Engineers, working on a program of twenty-four hours a day, consisting of two twelve hour shifts. Even under this pressure of speed, working day and night, excellent workmanship was accomplished, although at the loss of two lives by drowning.

All river piers are of the ordinary dumbbell type except the tower piers which are of the solid type with rounded ends and with the shafts partially hollowed out to reduce the dead weight. All piers except pier #8 on the south bank are supported on timber piles varying in length from 60 to 85 feet, all piles being driven to sustain a safe load of 45 tons each. The piers were designed to care for the dead and live load of the superstructure as well as a 30 pound wind load and ice conditions. Each river pier is protected with a steel shell between the elevations of minus two and plus twelve, or for a height of 14 feet, which adequately covers the range between tides; mean low tide being at elevation 1.7 and mean high tide at elevation 6.8. Figure 2 is a construction photograph taken on August 1, 1930, showing a layout of the piers as viewed from the Pennsylvania shore. The method of forming the shafts of these piers can be noted in the background of this picture.

The method of construction for building the piers was the same as generally used in open work. The location of all piers was made with the use of triangulation systems having base lines on each bank of the river and being so tied together as to form a check on each other. In addition to the triangulation, it was possible to check the pier locations by direct chainage between piers. This latter was made possible by the speed of the contractor whose equipment was located on as many as four piers at one time. A 700 foot tape, calibrated in the field for temperature and pull, was used in checking between the tower piers, a distance of 540 feet. This triangulation system was first used in locating the temporary pile docks. After these were driven, timber

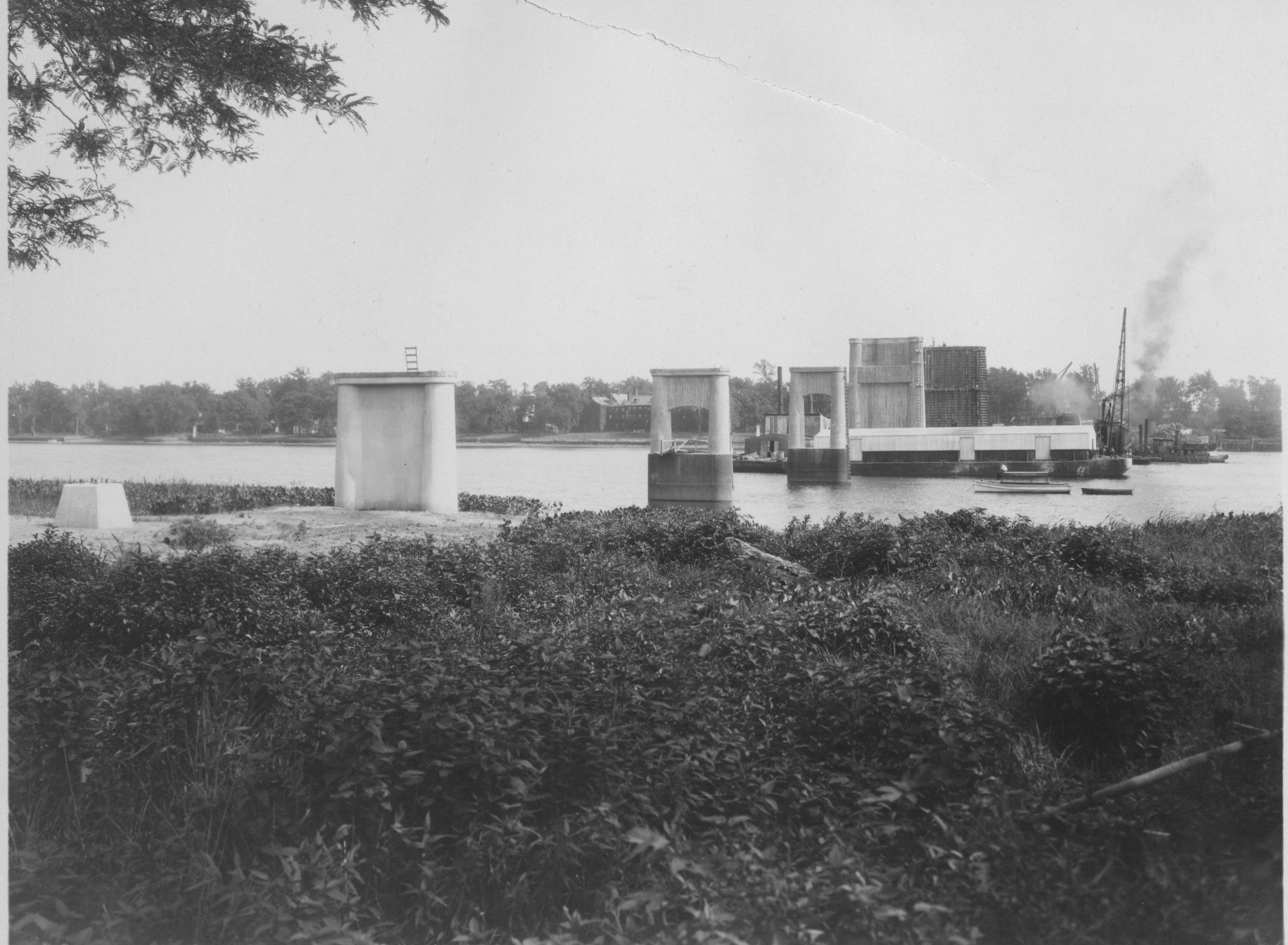


Fig. 2- Layout of piers from Pennsylvania shore. Pier #5 with forms.

frames were floated into position and anchored to these docks, their location being again checked from the triangulation system. Around these docks were driven the steel sheet piling forming the cofferdam, within which excavation was carried on by the use of clam shell buckets. This excavated material consisted of mud, sand, clay, gravel and boulders. After completion of the excavation, a thorough examination of the bottom was made by a diver, and upon a satisfactory report from him, the driving of the piling was started. All piers except the north shore pier rested on either sand, gravel and boulders, clay or gravel and clay, any of which would probably carry the pier loads without the aid of piling. Because of the alluvial nature of this part of the country, and the possibility of scour, it was decided to use timber piling to overcome this possibility. The piling used were of yellow pine with 8 inch tips and were driven with a steam hammer, supported in leads suspended from the boom of a derrick boat. These leads were set in position over the pile, supported at the bottom on timber struts in the cofferdam, and guyed from the top to the sheet piling forming the cofferdam, thus providing a rigid set of leads easily moved from pile to pile. Water jets were also used to secure the penetration desired. Both single and double acting steam hammers were used, the single acting hammer giving the best results. All outside rows of piling were first driven on a batter, their location checked

Fig.3- Steel sheet pile cofferdam at Pier #6 · Cutting off piling. Seal poured.

by a diver, after which the inside rows were driven. The spacing of piling was approximately 3 feet on centers. The underwater concrete seal was poured through a 12 inch steel tremie pipe in one single operation, the maximum time for any one seal being thirty-six hours. A 1 yard mixer was used, all work being carried on by use of floating equipment. This equipment consisted of four barges equipped with whirlers, two floating derrick boats, one cement boat, one pump boat, one mixer boat, and four miscellaneous barges for storing of equipment and materials. After the concrete seal had set up for a period of two days, the cofferdams were pumped out and the piers completed. Figure 3 is a photograph taken August 12, 1930, showing the piling within the cofferdam of pier #6. The seal has been poured, the water pumped out and the men are cutting off the piling and cleaning the mud and laitence from the concrete. The concrete mixes were proportioned by volume; those in the seals and coping being 1:2:4, while that in the balance of the piers was 1:2½:5. The shafts of the piers were formed and braced in the ordinary manner, using timber forms and steel tie rods, after which they were poured in either two or three lifts, depending upon the amount of concrete and the height of the lift.

Fenders for the main tower piers were constructed of creosoted timber, supported partially on the piers and partially on piling driven outside of the piers. The fenders are of heavy construction, being built entirely around the pier, in order to provide protection to themselves from ice conditions. Steel nose plates were used at each end and all fastenings were galvanized to care for the action of the salt air. Navigation lights are supported on the fenders and are made accessible for maintenance through ladders running down the piers from the truss spans above.

APPROACHES.

The retaining walls used at each end of the bridge are of reinforced concrete, designed as a standard cantilever wall with reinforcing in the rear face. The footings are reinforced, and are supported on a compact sandy material. The maximum bearing pressure is two tons per square foot. The fill between the walls is a sandy material, placed in layers thoroughly wetted and rolled, thus giving a very compact fill. The walls have battered faces with horizontal rustications thus giving them a very pleasing appearance. Both the walls and their concrete handrail were rubbed with carborundum bricks. Figure 4 shows the actual construction of the retaining walls while figure 5 shows the method of placing the fill, the photographs being taken during work on the Pennsylvania approach.

The roadway surface on these fills, having a maximum grade of 5.25% donsists of a reinforced concrete slab, designed on the standards of the New Jersey State Highway Department. Transverse joints and longitudinal joints along the curbs are provided with one half inch asphalt rubber fill-

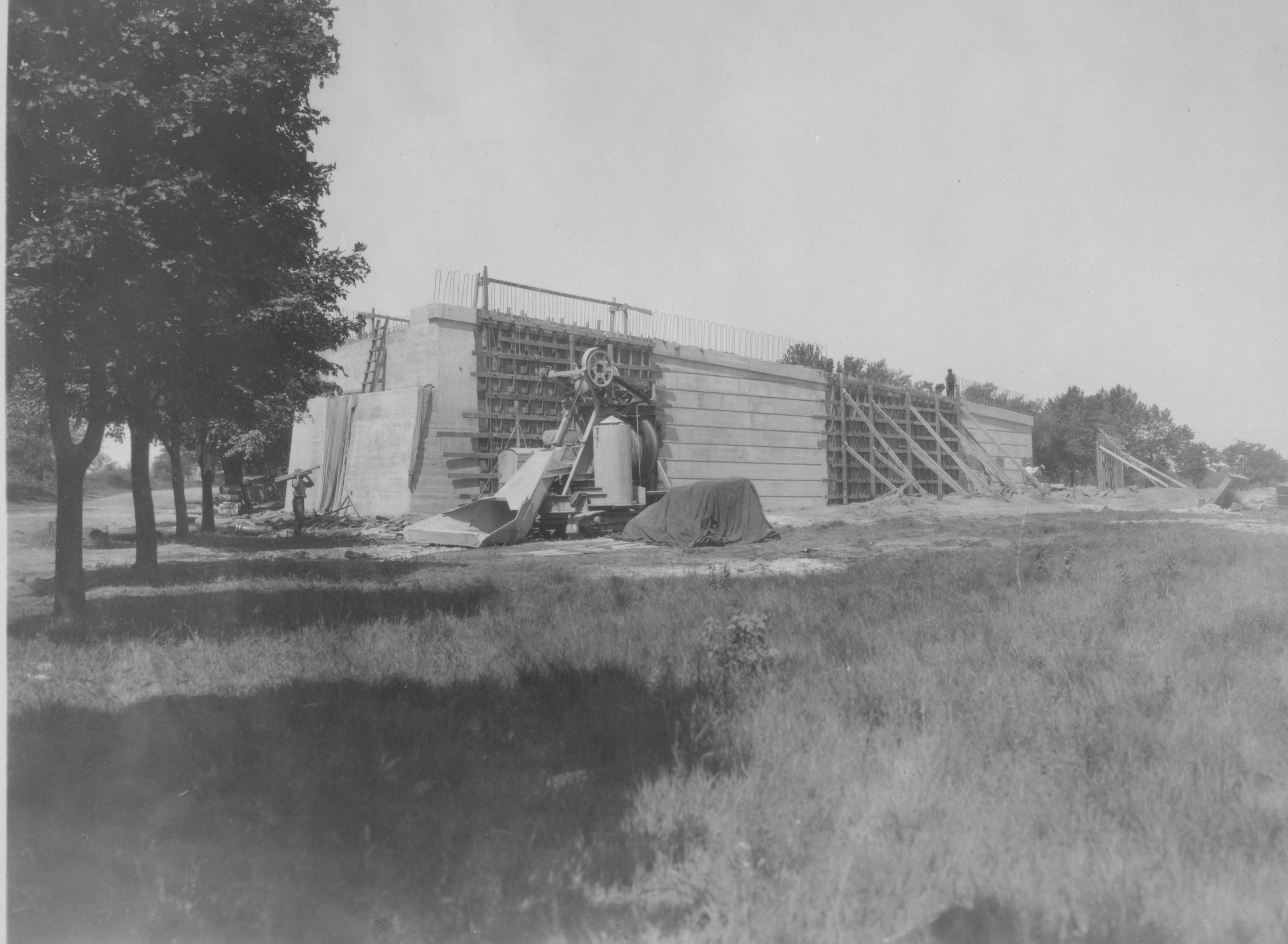


Fig. 4.- Retaining wall construction. Pennsylvania approach.

Fig. 5- Placing fill between walls. Pennsylvania approach.

er, while a keyed longitudinal joint is used in the center of the roadway and filled with asphalt. All joints are keyed and doweled to take shear, one end of the dowels being greased and arranged to permit sliding.

On the south approach approximately 700 lineal feet of steel beam spans are used, the span lengths being 35 and 40 feet. These lengths were determined partly by economical considerations and partly by column spacing required at the street and driveway intersections. A 21 foot transverse spacing of columns is used to care for a present 18 foot street over which the structure spans longitudinally.

This approach contains one panel of longitudinal bracing for each three span unit except on the lower units where the short columns themselves provide sufficient stiffness. Continuity in stringers over three panels is used to give a more rigid structure, thus avoiding excessive bending stresses on end connections, and minimizing the possibility of cracks in the concrete floor due to a tendency for the slab to take tension over columns in attempting to act as a top flange for the stringers. Expansion joints are also used at intervals of three panels to avoid large temperature stresses in the columns, although it was found necessary to use pins at the bottoms of the shorter columns.

In order to furnish the maximum longitudinal rigidity, economically, three longitudinal stringers are used, on which rests steel cross beams at about 3 foot intervals.

This layout also gives a very satisfactory and easy method of caring for the floor system around curves and on the super elevated portions. The floor slab is a 6 inch double reinforced slab, with no separate pavement being used or contemplated being used, as 1/2 inch of wearing surface is figured in the thickness used. The slabs were poured with the aid of vibrators on the forms, thus permitting a dry concrete to be used and one producing great density. The mix was 1:2:3½. The concrete was mixed at a central plant, hoisted to the deck and then delivered to the forms in cars and buggies. In temperatures below forty degrees, care was taken to see that the water and aggregates were heated and the concrete protected in general.

The columns for this approach consist of 14 inch rolled H beams, thus requiring very little fabrication, since they contain no lacing or tie plates. They are supported on concrete footings resting on a sandy material with a maximum bearing load of 1.6 tons per square foot. No special note need be made of the erection of the steel for this approach as it was erected with the use of a caterpillar tractor equipped with a derrick, much in the same manner usually employed for such work.

At this point it may be well to note the kind of paint being used. All metal received one shop coat of brown paint, applied by spraying. The field paint consists of two coats of aluminum paint applied by brushing, and applied to all steel except the inside of the handrails, which are painted a bright orange. To agree with the painting of the handrail, a 6 inch orange stripe is painted down the center of the roadway, thus dividing the traffic into its proper lanes.

The north approach, spanning the drive along the Pennsylvania shore, consists of a three span continuous deck girder approach laid out on a 525 foot radius. Stringers are eliminated to give simple framing details on the super-elevated curve. The reinforced concrete slab is carried on cross beams spaced about 5 foot centers supported on the girders. The girders are made continuous to give greater rigidity for the shallow girder needed to provide the necessary clearance over the roadway mentioned, and also to avoid intermediate joints in the roadway. The structure is supported on 14 inch H columns with pins at the bottom. The three span unit is anchored at the north abutment with expansion being provided at pier #1. The columns are fully braced transversely and are supported on concrete pedestals on wood piling. The erection of this girder unit was made with the caterpillar derrick which was later used on the beam spans of the south approach as just mentioned above.

Four 125 foot deck truss spans are used, three in the north approach and one in the south. The floor consists of a $9\frac{1}{2}$ inch double reinforced concrete slab supported on rolled floorbeams spaced about 10 foot centers, these beams

Fig. 6 - Pennsylvania approach - Slab forms - Concrete tower, cars and track.

resting directly on angle supports on the top chord of the trusses. The truss members all consist of rolled sections of constant depth I beams, thus having no tie plates or lacing. This type of truss is economical, easy to fabricate, erect and maintain, in addition to furnishing a substantial appearance. These trusses were erected complete on the shore and hoisted directly in place by derricks, land rigs being used for span #7 while floating derricks were used for the three spans of the Pennsylvania approach. After the individual trusses were set in place, the bracing and floor metal was added. The pouring of the concrete in the decks was carried out in the same manner as previously described. Figure 6 shows a photograph of the work being carried out, showing the dump cars and track, with the hoisting tower in the rear.

TOWER SPANS, TOWERS AND COUNTERWEIGHTS.

The design of these tower spans is no different from the design of any other simple truss except for a small deadload concentration at the point of the rear leg of the tower. Other outside loads are carried directly into the front tower columns and directly into the piers.

The floor on the 200 foot tower spans consists of a 6 inch reinforced concrete slab supported on steel cross beams spaced about 3 foot centers, being the same as used for the south approach.

The outline of the truss and its depths were selected in order to give a pleasing appearance, the top chord of the lift span and the two tower spans being placed on one long vertical curve over a distance of about 840 feet. No pins were used at the tower end of the span, a fixed connection being used instead. A large steel casting is used as a shoe for the tower column, and the truss members at the Lo point have their connection at the lower end of this tower column. This connection is so made and shop reamed as to reduce to a minimum the secondary stresses resulting at the connection, and at the same time provide an economical connection required by the large loads at this point.

At the request of the fabricator, the bottom chords were made with top cover plates and the bottom angles turned out and laced. This change was made in order to reduce shop costs on the chords, but the increased cost of floorbeam connections resulted in abandoning the use of similar sections on the lift span as was contemplated. However, after determining upon the method of erection of the lift span, it was found necessary to use a compressive section to care for the erection stresses, so nothing was lost or wasted by the change made at first.

Firgures 7 and 8 show photographs of the erection of these tower spans. Falsework bents of piling were driven under alternate panel points, and on these bents were set light steel bents used to support the steel work.

Fig. 7- Erection of New Jersey tower span.

Fig. 8 - Pennsylvania tower span and deck truss spans.

27

The erection started from the shore end, moving toward the river. The photograph shows the traveling guyed derrick used for this erection. This method of erection has been common on building construction for many years, but it is only in the last few years that it has been adopted for bridges, being introduced for this work by the American Bridge Company. It was the first attempt for the contractor here, and a little difficulty was encountered at first, but this was speedily overcome and very good results were obtained from its use. The steel was erected ahead of the derrick and supported on the falsework. After the erection of two panels, the derrick was moved ahead and the same process repeated. Camber blocking was not used as each falsework bent was equipped with wedge jacks which were used in raising or lowering the panel points as required. After the erection of the span, the camber was checked and riveting was started with gangs on each side.

The towers are designed to carry the vertical load of the lift span and the counterweights in addition to 25% impact from the moving load. The front tower columns are lengthened and cambered during erection so as to permit them to be vertical after full moving load is applied. Longitudinal girders were built into the bracing to provide supports for the transverse beams used during the construction of the counterweights and for use later if found necessary to replace the counterweight ropes. These beams have been moved back from under the counterweights and can be seen in the photographs of the completed structure. The tower columns are provided with guides used for the alignment of the span and counterweights. The guides for the span have been flared at the bottom, thus directing the span to an accurate seat at the end of travel. Figure 9 shows the method of erection of these towers by use of an A frame derrick carried up the back leg of the towers. It was hoisted from point to point by use of blocks fastened to the tower. The hoisting equipment is located on the deck of the tower span. Small A frame derricks were placed on top of the towers to aid in hoisting small miscellaneous material and equipment.

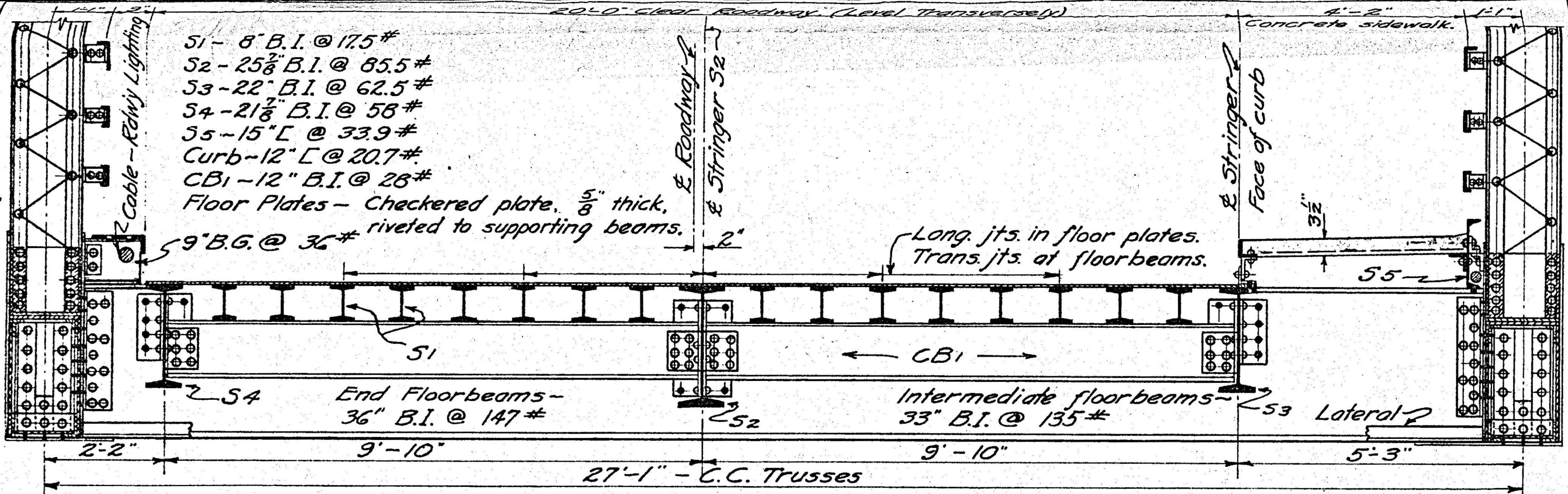
At the tops of the towers are found the large cast steel sheaves, 12 feet in diameter, over which pass the ropes connecting the span to the counterweights. These sheaves are pressed on their shafts with a pressure approximating 200 tons and are held in place with keys and screw dowels. The shafts, 19 inches in diameter, are of forged steel, and rotate in cast steel bearings lined with bronze bushings. The load on each sheave is approximately 650 tons. It is also of interest to note that large red beacon lights have been erected at the tops of the towers. These were placed at the request of the Department of Commerce, and will serve as a guide to aviation.

Fig. 9- Erection of New Jersey tower.

The counterweights are of concrete poured around a large steel frame or truss, fastened to the suspending ropes by means of large steel castings riveted to the frame and to which the rope sockets are fastened. The upper part of the ounterweights are made hollow to permit the placing of small concrete blocks, used to obtain a balance between span and counterweight. Thus with a perfect balance between span and counterweight, the work required to operate the span is reduced to a minimum.

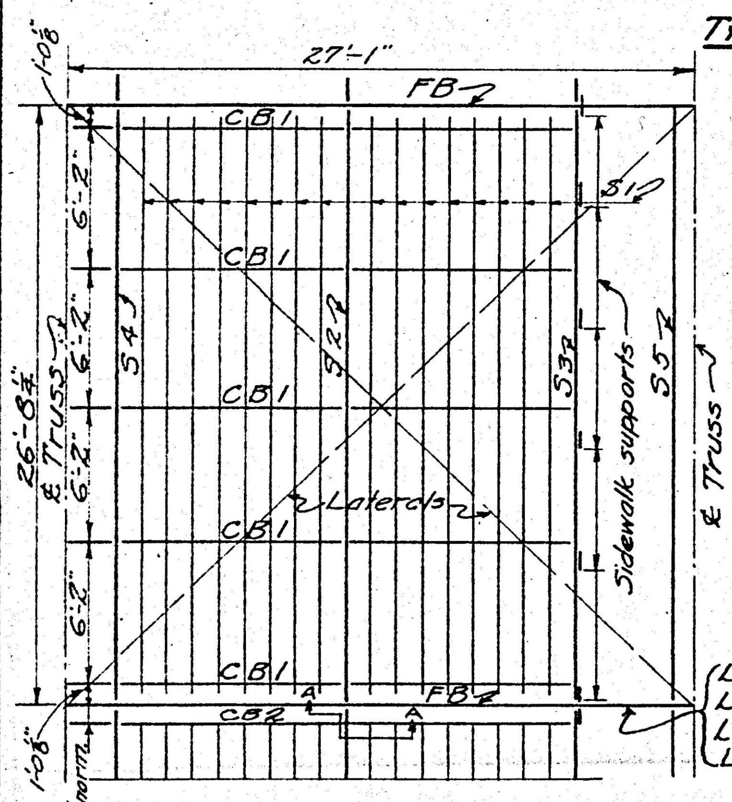
<u>LIFT SPAN.</u>

In the design of this span it was necessary to consider its total weight as probably being the governing factor in economy and usefulness. In order to keep the dead load to minimum, it was necessary to look into the type of floor system to be used. Every pound added to the weight of the floor meant an increase in the long trusses. Every pound added to the span meant added metal in the ropes, towers, machinery and concrete in the counterweights. Every pound thus added meant added loads to the piers, thereby increasing the number of piles and the sizes of the bases. Roughly, each pound added in the floor meant an additional cost of about 12 cents in the trusses, towers, piers and so forth. With this relation determined, it was possible to increase the cost of the floor itself in order to secure light weight and still secure economy in the total



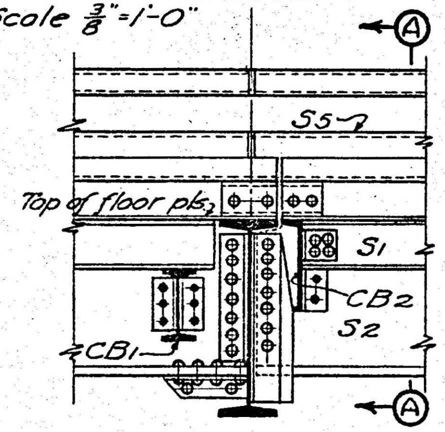
Typical Cross Section

Scale $\frac{3}{8}$" = 1'-0"



Marking Diagram - Typical Panel

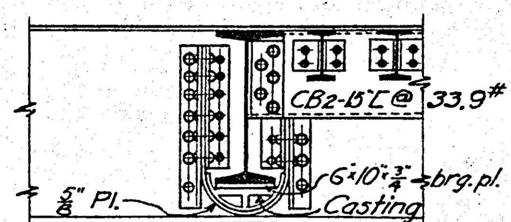
Scale - $\frac{1}{8}$" = 1'-0"



Longitudinal Section at Contraction Joints

At L4 & L8, L4' & L8'
Scale $\frac{3}{8}$" = 1'-0"

Fingered casting expansion joint at north end of lift span to provide for expansion of lift span. Sidewalk outside of truss in four center panels, and roadway widened to provide space for toll booth at center of span.



Section A-A

Burlington-Bristol Bridge

Across the Delaware River

Burlington, N. J. - Bristol, Pennsylvania

Details of Lift Span Floor

Ash-Howard-Needles & Tammen
Consulting Engineers

McClintic-Marshall Company
General Contractor

Fig. 10.

Sheet No. Y

M.B.M. 10-24-30.

structure. Whereas the cost of the floor adopted was about $12,000 more than the cost of a lightweight concrete floor with pavement; the total saving in the structure as a whole was estimated at $42,000.

A cross section of the floor used is shown in figure 10. It is composed of 5/8 inch steel traffic plates supported on longitudinal stringers spaced about 13 inch centers. Three main carrying stringers are provided, one at each curb and one along the centerline of roadway. Between these stringers are riveted transverse cross beams at 6'-2" centers and on which are supported the 8 inch stringers carrying the floor. A cross beam is used at each end of each panel in order to avoid riveting the top stringers into the floorbeams.

The floor plates were made in widths slightly over 3 feet and came riveted to the stringers in panel lengths. All rivets were flattened to about 1/4 inch. This floor differs from the Battledeck Floor, promoted by the American Institute of Steel Construction, in that only enough rivets are used between the stringers and the plate to hold them in close contact, no attempt being made to have the plates act as flanges for the stringers. The 13 inch spacing of stringers is used in order to avoid heavy bending stresses in the floor plates under heavy truck loads. With the arrangement used the maximum stress in the plate occurs with moderate sized trucks having narrow tires. Large trucks with wide tires will deliver a large part of their load directly to the stringers, thus producing smaller stresses in the plates. The plates are heavy enough to distribute some load to the adjoining beams, so that individual stringers are designed to carry only 80% of the maximum wheel load which is believed to be conservative.

Before adopting this steel plate floor, several other types were considered and given careful study. Of these, the most economical was a wood floor with asphalt plank pavement. Since it did not offer a fully fireproof construction and was not one of permanent character, it was decided that the additional expenditure for the steel floor was justifiable.

The floor plate is a standard traffic plate, the upper surfaces being deformed. Such plates are in use over wood floors on other bridges having similar climatic conditions, and have proven satisfactory. The grade of the floor was kept to a minimum to avoid slipping, the maximum grade being 1% except for a short distance at each end of the span where it approaches 3%.

The desirability of using an asphalt plank pavement on the plate floor was also considered, in which case it would have been necessary to use a smooth plate. Several installations of a similar character have been recently used, but due to the short period under which this has been tried, no accurate information is available as to its permanence, thus causing the idea to be abandoned. Then too, it has been the writer's experience that the asphalt plank is more slippery than the steel checkered plate.

Four intermediate floor joints are provided in the lift span floor, thus avoiding excessive bending stresses in the floorbeams caused by the stretch of the bottom chords under load. These bending stresses were further reduced by the adjustment in the lengths of the stringers which were not riveted until the span was swung.

At the expansion end of the floor, as well as at other expansion joints in the structure, tooth castings are used. This provides a stiff and rigid joint, eliminating all rattling and other troublesome features usually found where flat plates are used.

The design of the lift span itself is the same as for any other simple span, except that the reactions are considered as being at both the Lo point and the Uo point. Since the dead load weight of the span is balanced by the counterweights, the only reaction on the shoes, except during erection, is that of live loads. The reaction at the UO point is from dead loads with impact for moving. Because of the great length and height of span, the Warren type of truss was adopted, due to its appearance and ease of using sub-divided panels. In addition to the ordinary dead and live load concentrations, this span was designed for the weight of the houses, machinery, and so forth, that are located at the center of the span. This added weight has been

Fig. 11- Erection of lift span - Closure.

36

omitted on some of the earlier bridges built, by placing this operating equipment on the towers. This has not proved as satisfactory, due to the lack of synchronication between the operating units at each end of the bridge.

Figure 11 shows the closing of the lift span during erection and very clearly demonstrates the method used. It was first thought that this span should be built on barges off at one side of the bridge location, and later floated into place. It was impossible to erect the span in place on falsework, as no permit could be obtained from the Government to block the river channel for the length of time needed. Since the method of floating spans into place has become so prominent, it was natural that this idea would be first considered. Only after long discussion and figuring, was this idea abandoned, and then because of the great risk involved in its undertaking. The center of the span is about 66 feet above high water and the depth of the truss at this point is over 60 feet, thus placing the center of gravity of this floating mass more than 75 feet above the center of gravity of the floating barges. When one considers the overturning effect from wind, unequal loading and leaking barges, it is not hard to understand the thoughts and responsibilities of the Contractor, which caused abandonment of this scheme.

In the erection of large steel arches, the use of tie-backs, rather than falsework, has played a prominent part in the past decade. Its use was first employed on the Hell Gate arch at New York, and more recently on that great steel arch at Sydney Harbor, Australia. From this method of erection came the idea adopted for this bridge. Instead of having to build large and heavy back-stays or buttresses, the spans themselves were used, the supporting cables being carried over the large sheaves at the tops of the towers and fastened to the hips of the tower spans. The decks of these latter spans, as well as the counterweights, were poured in order to provide sufficient weight to resist the uplift from the cantilevered portion of the lift span. This also placed tension in the bottom chords which reduced the compressive stresses placed therein by erection. The method of erection used is entirely new and original in the construction of movable bridges, and much credit should go to the Contractor devising the scheme.

The lift span was erected about 1½ feet above its final position on the piers. This was done to allow for the stretch in the suspending ropes in finally swinging the span, and also to provide adequate space for jacks to be used in raising and lowering the ends of the span as desired. The erection was started at the Lo point of the north end of the span. Both towers and the lift span had special gusset plates at this point, these plates extending beyond their normal position, and being bored for 8 inch pins. The north half or end of the trusses was erected first, the cantilevered section being supported by the tie cables running up over the sheaves and tied back to the tower spans as previously mentioned. These tie cables were $1\frac{3}{4}$ inches in diameter and "pre-stressed", that is, stressed to a point where nearly all the normal first stretch is eliminated, and then accurately measured. Their fastening to the trusses was by pins through special plates built into the trusses and later burned off. These fastenings were made adjustable through special arrangement of screws, thus permitting the various parts of the cantilevered sections to be raised and lowered at will, and also providing a means of adjustment for unequal stretch and measurement of various ropes. A larger number of ropes was used as the erection proceeded outward. On the north half, cables were used at the U2 and U4 points only, this section being supported on falsework at L6, the only falsework used in this erection. This gave a clear horizontal width of channel of 350 feet during erection and met the requirements of the Government. After landing on this bent, the cables were removed and used on the erection of the south half of the span, the north half being continued by direct cantilevering without the use of cables. This was possible, since the reactions of the span were carried on the falsework and on the 8 inch pins at the Lo point. The cantilevering of the north side was carried out to the L8' point or for a distance of 12 panels. The south half was carried out by cantilevering and suspending from the cables until the closing point was reached, or for a distance of 8 panels. The erection of the steel work was accomplished with the guyed derrick described in the erection of the tower spans. The derricks can be clearly seen in the photograph showing the erection of the span.

It will be of interest to note some of the means of adjustment used in keeping the span in alignment and in the closing. The span was connected to the falsework bent at L6 by means of a structural shoe bolted to the bottom chord and connected to the falsework bent with a 6 inch pin. This steel bent was set on wedge jacks resting on the timber falsework and were used to maintain the proper elevation at L6. Before the closure was made, hydraulic jacks were placed on the piers directly under the bottom chords, and the gusset plates at the north tower were burned away, thus cutting the span free at this point. The movement of these jacks, with the falsework bent as a pivot, raised and lowered the end of the cantilevered portion to its proper vertical position for closure. Small jacks and steamboat ratchets moved the span longitudinally in either direction desired to bring the joint into line.

In addition to the vertical and longitudinal adjustment, it was necessary to care for about 3 inches of lateral or transverse misalignment. This too was accomplished partly by use of cables and ratchets as temporary laterals, so arranged to pull against the long laterals; partly by crowding one truss with jacks; and partly by closing with a slight kink in the span which of course came out when the span swung free. The entire adjustments required for closing were made in one day. As soon as the closure was complete and fitted up, the span was swung by jacking at the Lo points on the piers until the span was free from the falsework bent. It was then connected to the counterweights by the connecting cables and the span jacked down until the entire mass of span and counterweights was all free from supports, the entire load being taken in the tower columns.

WIRE ROPES, OPERATING MACHINERY AND EQUIPMENT.

The principal element of the vertical lift bridge is a simple span equipped with machinery for operation, suspended at the four corners with wire ropes which pass over sheaves at the tops of towers and connect to counterweights, approximately equal to the dead weight of the span. The operation is through machinery comprising drums operated by motors or gas engine, these drums controlling the operation of the ropes. Each drum contains and controls two pairs of operating ropes which pass, respectively, over and under deflector sheaves at the corners of the span, thence one pair upward and one pair downward to connections on the tower and the pier. Revolution of the drums in one direction winds on the up-haul ropes, pays off the down-haul ropes, and lifts the span. Reversal of the drums lowers the span. The operating ropes are 1 3/8 inches in diameter and consist of 6 strands of 37 wires each. These ropes are made with a hemp center, the large number of wires being used because of their greater flexibility. The operating ropes connect to take-up devices and may be adjusted to have the same general tension at each corner. The span is thus moved, held level during operation, and may be held at any point by stopping machinery. No other locks are used. Sixteen 2 inch diameter ropes per corner connect the span and the counterweights, each rope consisting of 6 strands of 19 wires each and with a hemp center and 6 filler wires. Figures 12 and 13 are photographs of the span closed and fully opened.

The operating machinery consists of the ordinary type used with spur, gears and pinions, assembled in a cast steel frame, so as to be rigid and provide good alignment. The gear frame is joined by transverse shafts to the operating drums set outside the truss members. The motors and gas engine are also directly connected to this gear frame, all connections between motors, gas engines and shafts being made with large flexible couplings to care for improper alignment and deflections.

The operation of the span is by two 80 HP electric motors which raise or lower it a total distance of 74 feet in two minutes. In case of failure of power, an auxiliary 75 HP gas engine has also been provided. This auxiliary operation will operate the span full height in seven minutes under ordinary conditions and in ten minutes under conditions

Fig. 12- Lift span in closed position.

Fig. 13 - Lift span in open position.

of ice and wind. Both motors are equipped with electric hydraulic brakes and in addition, a hand operated service brake is provided. A cut-outsswitch is provided so that it is impossible to operate the span by electricity while the gear train for the gas engine operation is thrown into position. This prevents the tearing to pieces of this set of gears which operate at a much lower speed than those for the electric operation.

Electric power for operation is brought to the span by the use of trolleys on the face of the tower, while the current for lighting as well as the telephone circuit, is brought to the span by the use of flexible cable. Electrically operated traffic gates are provided on the approach spans, each gate being equipped with warning gongs and traffic lights. These gates are electrically interlocked with both the motors and the gas engine so that the span cannot be operated until the gates are fully closed. Limit switches have been provided so that the power is cut off from the span at points 6 feet above and below the extreme closed and open position of the span. These switches are provided as means of safety in stopping the span should it become out of control of the operator. Under safe and normal operation it is possible to by-pass these points so that the power will not be completely cut off, but will be reduced to only 35% of its full amount.

The toll collector's booth, the office and machinery

Fig. 14 - Center of lift span showing houses.

Fig. 15 - Longitudinal view of lift and tower spans.

houses are all located at the center of the lift span. Due to the infrequent number of operations of the lift span, it is expected that the toll collector will serve as operator, thus making it desirable that these houses be close to one another. The toll booth is in the center of the roadway, which is widened at this point to provide for two 9 foot lanes of traffic, one on each side of the toll house. The sidewalk is carried on the outside of the east truss at this point while the office building is on the outside of the trusses on the west side. Figure 14 shows the house, while figure 15 is a view looking toward the center of the lift span, being taken at the end of the New Jersey tower span.

ORGANIZATION.

The bridge is owned and will be operated as a toll structure by the Burlington-Bristol Company, a private corporation. It is extimated that the cost of the structure complete, without approach roads, will be $1,250,000. The bridge was designed for the owners by Ash-Howard-Needles & Tammen, Consulting Engineers, at Kansas City and New York. The writer was in charge of construction for the Consulting Engineers. The general contract for the construction was given to the McClintic-Marshall Company of Pittsburgh, who are to be commended upon the rapid and satisfactory completion of the structure.

BIBLIOGRAPHY.

The data for this article was gathered by the writer during the past year and a half, during which time he has been connected with both the design and the construction of the Burlington-Bristol bridge. Acknowledgement is given to the writer's firm, Ash-Howard-Needles & Tammen, Consulting Engineers, for the use of drawings, calculations, specifications, and records, all of which were used in the preparation of this thesis.

INDEX.